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PRACTICAL DESIGNING

OF

RETAINING WALLS.

BY

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PRACTICAL DESIGNING
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INTRODUCTORY.

In designing masonry works there is hardly any subject that presents itself more frequently than the retaining or revetment wall; and in some form or other it is found to enter into almost every design. To the military engineer no less than to his civil brother is the subject one of importance and interest, forming as the revetment wall does, for the most part, a component element in works of defence. To military engineers in truth is due some of the most valuable information that civil engineers possess regarding the theory of earth pressure, and although further considerations are involved in designing revetments for military works than the mere support of earthwork, there is still to be derived from the experiments and researches of military men information of much value to civil engineers. The subject is one that has received the fullest and most able treatment at the hands of mathematicians, and solutions for every case that could possibly occur in practice are to be found in our text-books. But the mathematical investigations of this and many other questions of common occurrence in practice, unquestionably valuable as they are, in determining the principle involved, and establishing final rules applicable to practice, are, it is believed, but rarely resorted to by practical engineers. Even when such examples have to be dealt with by those sufficiently acquainted with the mathematical mode of proceeding, they are generally solved without hesitation by some empirical rule, derived from experience. Such a method may, and doubtless occasionally does, lead to accident from weakness, and not unfrequently to clumsy waste of material and consequent expense. But it is not clear that less of failure or clumsiness would result if every retaining wall were calculated with mathematical precision, for in truth the data involved are so variable and imperfect, and the disturbing causes are of such a character as to neutralize to a great extent the accuracy of the investigation. With certain specific data theoretical accuracy can always be attained; but the engineer as a rule knows nothing with absolute certainty either of the weight of the earth he has to sustain in position, or of the masonry that he intends to adopt in doing so. These and other data he must assume before he enters on his calculations; and though there is not in these, as in many other investigations, any necessity to attempt an extreme degree of refinement, which would be inapplicable for every-day practice, yet there can be no good excuse for dealing with the matter by hap-hazard and guess-work.

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* Practical Designing of Retaining Walls, by Arthur Jacob, A. B.

It is not proposed now to regard with

more than a cursory glance the principles involved in determining the strength of walls to support earthwork. Such simple rules will be given, as it is hoped will serve—due regard being had to the peculiarities of each particular case—to guide the less experienced in designing works of this class. The empirical mode of dealing with the question is clumsy and unscientific, whilst the formulæ usually given are so complicated as to render their application to practice out of the question.

SPECIFIC CAUSES OF FAILURE.

It must not be presumed that the failure and destruction of a retaining wall is necessarily due to the wall being of itself insufficiently strong. It may be quite heavy enough to resist the pressure of a bank, if due regard be had to the mode of forming the earthwork, and to drainage; but if these points be not fairly considered and observed at first, a retaining wall of quite sufficient thickness will probably give way sooner or later. As much care should in fact be devoted to the method of backing up and draining a wall, as to the calculation of its section; for indeed if these matters be disregarded, no retaining wall, properly so called, can be implicitly relied upon to stand. With the exception of one particular case, which will be noticed hereafter, walls are designed on the assumption that they are to support a dry material—or one, at any rate, not permeated by water—and that the material is to be deposited in such a manner as to have no predisposition to slide against the wall. It is, of course, also presumed that the wall shall be of fair workmanship and materials, and where these points cannot be relied upon, as is sometimes the case, especially in foreign works, some allowance should be made in the dimensions of the wall. It has not unfrequently happened that a retaining wall will have stood for a considerable number of years without showing any appearance of yielding, and yet will give way suddenly and completely, without apparent cause. Such failures can generally be accounted for by the fact of the wall not being designed to resist a maximum pressure, and never having been tried fully till the time of its destruction. Much apparent anomaly is observed in the way that retaining walls are found to fulfil the purpose for which they are designed: for whilst some will yield, others of less dimensions will continue to stand; such apparent inconsistency giving occasion for ingenious theories, most of them entirely unsupported by fact or experience. The truth is, that imperfect drainage, defective foundations, or rotten work will account for almost every conceivable case of failure.

FIRST CASE—HYDROSTATIC PRESSURE.

The first and simplest case of a retaining wall to be considered is that in which the pressure of water has to be counteracted; not indeed that the question in such a form belongs strictly to the subject under notice; but it nevertheless becomes absolutely the method of determining the strength of walls for certain positions. It not unfrequently happens, as in some hydraulic works or with the wing-walls of aqueducts, that the infiltration from behind, which is not always avoidable, may produce such a pressure as no retaining wall properly so called could be expected to bear. With this view the engineer's limit of safety will be attained when the structure is designed to sustain the full hydrostatic pressure. The pressure of water upon any plane surface immersed is known to be *equal to the area of that surface, multiplied by the depth of its centre of gravity below the level of the water, and by the weight of a unit of water.* Generally speaking the unit adopted in calculation is a foot; and the unit of water being taken at a cubic foot, weighing 62.5 lbs., the resulting product, from the multiplication of the three quantities, will give the *pressure* in pounds on the surface immersed. Let it be supposed for simplicity that water to the depth of 10 ft. has to be sustained by a vertical rectangular wall. It is usual to take but 1 ft. length of the wall for the calculation, though it will not affect the result whether 1 ft. or 100 ft. be the length assumed. We then have the surface under pressure = 10 sq. ft., the depth of the centre of gravity = 5 ft., and the weight of a cubic foot of water = 62.5 lbs.; the product of which quantities give us 3,125 lbs., the *pressure* on 1 ft. length of the wall. But this pressure is not the whole of the force that the wall has to resist; the leverage that it exerts must also be taken into account. In the example under consideration—namely, that of a vertical plane, with one of its sides coinciding with the surface of the water, as in Fig. 1.—the whole of the pressure is so distributed as to be equal to a single force acting at a point one-third of the depth from the bottom. Thus the total force to be resisted by the wall is 3,125 × 3.33 = 10,416, which is the *moment* tending to overturn the wall.

MOMENT OF RESISTANCE TO OVERTURNING.

It is evident that a certain weight of wall must be opposed to this overturning force; and as the height of the wall and the length are determined quantities, the thickness alone remains for adjustment. But as a rectangular wall in upsetting is considered to turn upon a single point, F, Fig. 1.—namely, the outer line of the foot of the wall, there will be a certain amount of leverage to assist the wall in resisting the pressure of the water. This leverage is the horizontal distance of the centre of gravity of the wall from the turning point F, and when the structure is rectangular and vertical, it is equal to half the thickness. *The amount of the wall's resistance will then be equal to the number of cubic feet in one foot of its length, multiplied by the weight of a single cubic foot of masonry, and by half the thickness of the wall.* Taking w = the weight of a cubic foot of water = 62.5 lbs., w^1 = the weight of a cubic foot of masonry, say 112 lbs.; x = thickness of the wall, and h = the height; the condition of simple stability will be fulfilled when

$$w^1 \times h \times x \times \frac{x}{2} = w \times h \times \frac{h}{2} \times \frac{h}{3} \qquad (1)$$

$$\frac{w^1\, h\, x^2}{2} = \frac{w\, h^3}{6}$$

and solving for x we get

$$x = \sqrt{\frac{w^3\, h^2}{3\, w^1}} \quad . \quad . \quad . \quad . \quad (2)$$

The thickness of the wall = 4 ft. 4 in.

EXAMPLE.

A simple example has been selected for illustration, but of course a rectangular section of wall would not be found generally applicable in practice, nor would it be expedient to limit the dimensions of a retaining wall of whatever kind to the minimum that would sustain the pressure; some margin of safety must therefore be allowed, to cover inferior work and materials. It is true that no account has been taken of cohesion, which, if the wall be founded on rock or concrete, may be assumed to add to its stability about 7,000 lbs. for every square foot of base. In addition to this, practice seems to indicate an increase on the calculated thickness, and in the example the mean width might be augmented to 5 ft., the stability being further increased by altering the section from a rectangle to a battering wall with offsets at the back.

A good general rule for the dimensions of a wall designed to support water or earth in a semi-fluid condition will be—

Top breadth = 0.3
Middle do. = 0.5
Bottom do. = 0.7

The height being represented by unity.

Proceeding to the consideration of walls for the support of dry earth, it will be found that the question is one that will in general require the engineer to exercise his judgment, to determine what angle of repose he will base his calculation upon. The natural slopes assumed by earths of different tenacity are so various, that an average figure cannot be adopted with safety; the calculation of pressure from earth, in fact, depends essentially on this point, and a disregard of it will lead to very doubtful results. The following are a few of the slopes assumed by different materials, but it is probable that the engineer's judgment will be of more service than any table in deciding the angle of repose. The examination of a district in which works are intended to be carried out will always suffice to satisfy the designer of the nature of the material that he is dealing with, and may enable him to proportion his works very nearly to the requirements of safety and economy:—

	Angle of repose.	Slope.
Dry sand, clay and mixed earth	From 37°	1.33 to 1*
	to 21°	2.62 to 1
Damp clay	45°	1 to 1
Wet clay	From 17°	3.23 to 1
	to 14°	4 to 1
Shingles and gravel	From 48°	0.9 to 1
	to 35°	1.43 to 1
Peat	From 45°	1 to 1
	to 14°	4 to 1

To which might be added as a special feature London clay; it appears under the influence of weather to be exceedingly unstable, slipping away to almost any angle of repose.

THEORY OF EARTH PRESSURE.

It has been ascertained by M. Prony that when a vertical wall sustains the pressure of a bank of earth the top of which is horizontal, the maximum horizontal pressure to which the wall can be subjected will be reached when the plane of fracture of the earth bisects the angle that would be formed were the earth to slope from the foot of the wall backwards at the natural inclination. This fact is somewhat striking, for it would appear at first sight, and was for long assumed, that the angle of fracture ought to coincide with the natural slope of the earth; such is, however, not really the case. If we suppose the angle made between the sloping plane and the vertical to be bisected, the prism of earth enclosed between the bisecting plane and the wall will represent the mass, the pressure of which has to be resisted; and this being the *maximum* pressure that a horizontal topped bank is capable of exerting, it is usually the point to be determined.

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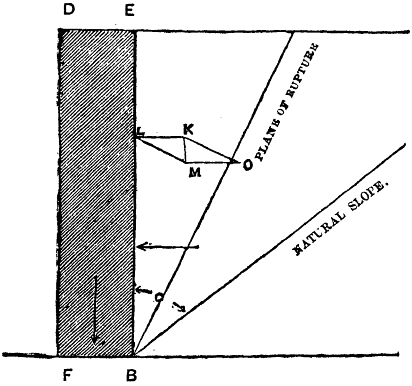
* Rankine's "Manual of Civil Engineering."

Referring to Fig. 1, the principle of earth pressure will readily be understood. Supposing the plane of rupture to bisect the angle *c*—which will be the case when the pressure is a maximum—the prism cut off will be the whole weight that the wall will have to sustain. Taking this prism for a single unit of length or thickness, the superficial area will represent the cubic contents. But the area of the triangle, taking *h* as the height of the wall, will be

$$\frac{h^2 \tan. \frac{1}{2} c}{2}$$

c being the angle contained between the natural slope of the earth and the back of the wall. It is only necessary to multiply this value by w, the weight of a cubic foot of the bank, to get the total weight of the prism.

FIG. 1.



This prism of earth is then like any other body resting upon an inclined plane; which in this case is the plane of rupture. It is sustained in position by the wall on one side and by the fixed portion of the bank on the other; and may be regarded as a solid mass of material without motion amongst its parts. The line K M represents the direction of the force of gravity, and the lines K L and K O the pressures exercised against the wall, and the force of the bank respectively. These pressures produce a certain amount of friction against the wall and the bank, but, as the friction against the wall does not materially affect the question, the friction of the bank alone is considered, and taken into account in arriving at the following formula which applies to the case of a vertical wall supporting a bank with a horizontal-topped bank:—

$$P = \frac{w\,h^2}{2} \tan.^2 \tfrac{1}{2}\,c \quad . \quad . \quad (3)$$

Having calculated the *pressure* of the earth, the next step will be to determine its *moment* to overturn the wall, and this can be ascertained, as in the case of water, by multiplying the pressure by one-third of the wall's height. This having been determined the next consideration will be, what weight of wall will suffice to sustain it; and the method of arriving at this is similar for the most part to that adopted for water. Taking, as above, the moment of the wall to resist the pressure, the following equation will represent the conditions of stability:—

$$\frac{w' h x^2}{2} = \frac{w h^2}{2} \tan.^2 \tfrac{1}{2} c \, \frac{h}{3}$$

And solving for x, the thickness of the wall, we have—

$$x = \sqrt{\frac{w h^2 \tan.^2 \frac{1}{2} c}{3 w'}} \quad . \quad . \quad (4)$$

If the weight of a cubic foot of earth be taken equal to a cubic foot of the wall, the value will be—

$$x = \sqrt{\frac{h^2 \tan.^2 \frac{1}{2} c}{3}} \quad . \quad . \quad (5)$$

which would give a thickness of 2.69 ft. for a rectangular wall of 10 ft. high supporting a bank of earth, the angle of repose being taken at 40 deg. The average weight of brickwork and ordinary clay will generally be nearly the same; but if great accuracy be desired, and the respective weights of the materials be known, the general formula No. 4 must be used.

The following table gives the weight per cubic foot in pounds avoirdupois of such materials as come under our consideration in solving questions relative to retaining walls:

	Weight of a cubic foot in pounds.
Sand—damp	120
Do. dry	90
Marl	100
Clay	120
Gravel	125
Brick	130
Brickwork	112
Masonry	130
Mortar	110

PARTIAL RETAINING WALL.

Having so far considered the first two cases, namely, those of a wall supporting a horizontal-topped bank of earth in a semi-fluid condition, and also in a state of comparative dryness, the next example that suggests itself to our notice for examination is that of a partial retaining wall, or a wall from the top of which the bank slopes away for a certain height—called the surcharge —either at the natural slope of the earth or at a less inclination. Such mode of construction is of very common occurrence, dwarf walls being frequently adopted on railway works where the cuttings or embankments are of considerable height, and when carefully designed are found to effect a saving of expense, both in construction and in the item of land. In cuttings the walls are carried up to such a height as economy dictates, and the slope is then trimmed back at the proper angle. Similarly with embankments, the walls are so disposed as to cut off the foot of the slope. In either case a little consideration will suffice to show whether the saving of earth and land area will cover the cost of the retaining walls. In military works, as well as civil, the partial revetment is very commonly used, being, indeed, a component part of almost every system of fortification.

The first particular case belonging to this class, though not of the commonest occurrence in civil practice, is when a partial retaining wall supports a bank, the face of which slopes back at an angle less than the natural slope of the earth. As M. Prony's rule, that the plane of rupture bisects the angle between the natural slope of the earth and the back of the wall, only holds good when the surface of the bank is at right angles to the plane of the wall, another mode of determining the angle for the maximum pressure must be resorted to. The simple construction given in the note enables us to arrive at the maximum pressure for a wall at any given batter, with the surcharge above sloping at any inclination. The equation arrived at is the expression for the maximum horizontal pressure:

$$P = \frac{w\,h^2}{2} \times \frac{\tan.\ \theta \tan.^2 (c - \phi)}{\tan.\ \theta - \tan.\ c} \quad . \quad . \quad (6)$$

the angle c being that between the back of the wall and the natural slope; $\theta =$ the angle made by the face of the bank with the plane of the wall; and $\phi =$ the angle between the plane of rupture and the back of the wall. The value for $c - \phi$ will be found in the note. Taking, for example, a vertical wall of 10 ft. high, supporting a bank that slopes back at an inclination of 20 deg. with the horizon, the natural slope being 40 deg., the value of tan. $(c. - \phi)$ will be

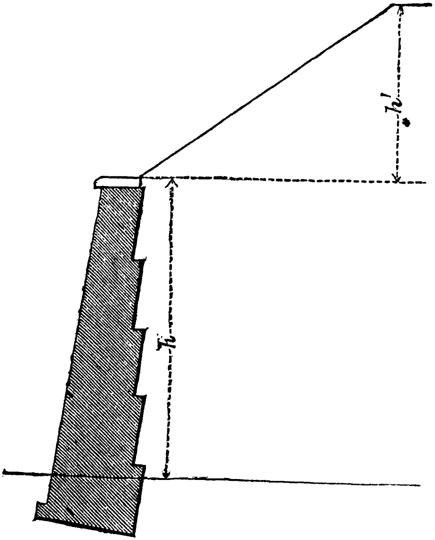
.4610; inserting this value and working out the equation, we arrive at a pressure of 2,100 lbs. against the back of the wall.

For the case of a revetment sustaining a surcharge the centre of pressure will be, as in the former case, at one-third of the height of the wall, giving a leverage of 3.33 feet. This gives 2,100 lbs. × 3.33 = 6,993, the moment of the earth tending to overturn the revetment. Equating this value to the moment of the wall, taking the cube foot of brickwork at 112 lbs., the same weight as the earth, and solving for x the thickness, we find it to be 3.53 ft.

DEFINITE SURCHARGE.

The next case to be considered is one of much more frequent occurrence in practice than that just mentioned; it is a partial retaining wall supporting a surcharge of earth, sloping away at the *natural* inclination, and terminating in a horizontal plane above. Cuttings and embankments partly supported by masonry works furnish familiar examples of this, which is denominated the "definite surcharge." The most convenient method of determing the thickness of wall in this instance will be to consider, first, the conditions of stability for an infinitely long slope, which, however, can only have a theoretical existence; and having arrived at the thickness of wall necessary to support such a bank, a simple reduction will give

Fig. 2.



the thickness required when the length of slope is limited.

It has been mentioned that when a vertical wall sustains a bank with a horizontal top, the plane of rupture for the maximum pressure is found to bisect the angle between the natural slope and the vertical. It is also an ascertained fact, that as the angle of the surcharge increases, the angle ϕ, or that between the plane of rupture and the back of the wall, also increases; until the face of the bank slopes at the natural inclination of the earth, and then the plane of rupture becomes parallel to it. From this it would appear that when the slope is infinitely long—a condition that could not exist in practice—the pressure will also be infinitely great; but such is not really the case. The ratio of the pressure of a bank, whatever its inclination, to the pressure exerted by an embankment level with the top of the wall can never exceed 4 : 1. The formula, then, for finding the maximum horizontal pressure exerted by an infinitely long slope against a vertical wall will be—

$$P = \frac{w\,h^2}{2} \sin.^2 c \quad . \quad . \qquad (7)$$

the notation being exactly the same as in the other cases. If we work this pressure out, using the same values for w, h, and c, as taken above, we shall find P=3,281 lbs.

Now for the leverage: we have, as in every other case, simply to divide the height of the wall by 3, which in our example gives 3.33 and the moment to overturn the wall = 3,281 × 3.33=10,925. Proceeding in the same manner as before, the width of a wall of brick to counterbalance an infinitely high bank sloping at the natural inclination, will be found to be 4.43 ft.

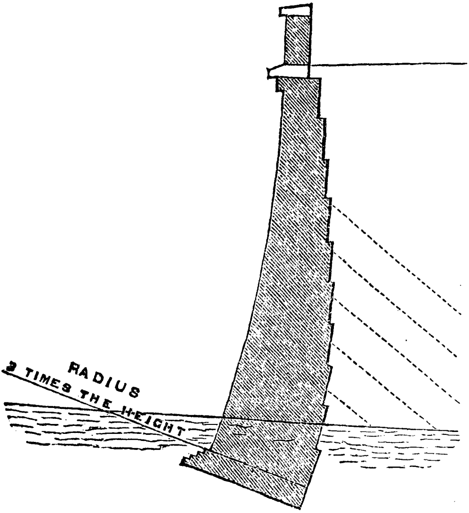
When the surcharge is very high as compared to the height of the wall, no reduction of the thickness will be necessary, for practically the slope may be considered infinite; but when the bank does not overtop the wall by a great height it will be well to apply the following formula to ascertain the corrected thickness. Let h= height of wall=10 ft., h'=height of surcharge above the wall, which we shall take at 20 ft., t=thickness of wall to support a horizontal bank, as found in the first case =2.69 ft., T=the thickness of a wall for a 20 ft. surcharge, t'=thickness for indefinite slope as found=4.43. Working this out the thickness is found to be 408 ft.

$$T = \frac{h\,t + 2\,h'\,t'}{h + 2\,h'} \quad . \quad . \quad . \quad (8)$$

So far we have considered the cases of more usual occurrence in practice, namely those in which the back of the wall is vertical or stepped, which is practically the same thing. For the calculation of leaning walls the reader is referred to the general formulas (A) and (B) given in the note; from the latter formula the horizontal resistance of any bank, supported by a wall at any angle of inclination, can be ascertained, and the leverage being in every case taken at one-third of the height of the wall, there will be no difficulty in designing a wall of such a section as will resist the pressure of the bank effectually.

The point to be kept in view is the moment of the wall, and this must be made to *exceed* the maximum overturning force of the embankment. It will not suffice to *equalize* the moment of the earth's force to the resistance of the wall, as has been done

FIG. 3.



in the examples above; a certain excess of resistance will be necessary, and this can easily be attained by giving the wall a batter, or else sloping it back so as to throw the centre of gravity of the mass as far back as possible, in a horizontal direction from the outer line of the foot of the wall. The line of the centre of gravity must not, however, be allowed to fall inside the base of the wall, otherwise the stability of the structure will become dependent on the support of the bank, and will have a tendency to slide away from its position.

It has been stated, and taken for granted, that banks of earth, when they destroy retaining walls, do so by turning them over; this is, however, not invariably the case. It has occasionally happened that walls have been moved bodily forward, sliding on their base. Such an occurrence is certainly accidental, and is probably the result of the wall having been founded on an unstable material, perhaps on an inclined bed of moist and uncertain soil. Walls have also given way in rare instances by the upper courses of the structure yielding to pressure, breaking off and falling over; a contingency that is probably due to the upper part of the bank becoming suddenly charged with water, and exercising an undue pressure on the wall before there is time for the water to drain away. These must be regarded as rare contingencies, arising out of some defect of the foundations, or backing; and cannot affect the consideration of the wall's stability generally. The theory of the wall being turned over on its base provides for the greatest trial to which the structure can be subjected, or, in other words, the wall would as a general rule give way under a much less pressure by falling over, than would be required to overcome friction, and move the wall forward in its entire state; if therefore the structure is considered as having to withstand the overturning force, it will always be strong enough to resist being pushed forward.

RETAINING WALL WITH CURVED BATTER.

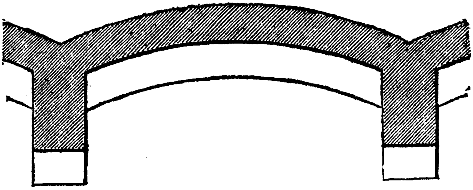
A form of retaining wall commonly met with in practice, especially in brickwork structures, is that with a curved batter, stepped in offsets at the back. The curve usually adopted is the arc of a circle, the radius of which is from $2\frac{1}{2}$ to 3 times the wall's height; and the centre of the curve is as a rule in the same horizontal plane as the top of the wall. In such structures the courses are made to radiate from the centre, and the result is that the joints of the brickwork at the back are thicker than is either necessary or advisable. When the radius of curvature is large, the increase of thickness is inconsiderable, but it becomes decidedly an objection when the curve is a short one; for the thickness of the wall will not become reduced in the same proportion as the height or as the radius of curvature. The dimensions of a wall of this kind may be determined with sufficient accuracy, by first considering it as a leaning wall at a given slope, and using the general formula (6), and in this manner a very close approximation to the thickness may be arrived at. There are, it is true, specific formulæ given by some authors for determining the thickness of curved walls, but they are too complex for application in practice. The effect of the curvature will be to add to the wall's stability by bringing the centre of gravity farther in towards the bank, and this, indeed, is the only advantage that the curved form possesses; it is difficult to construct, and consequently expensive; for the saving of material, if any, is very trifling. In architectural effect it certainly has no advantage over the wall with a straight batter, for the simple reason that it does not convey the same idea of strength. If the curved wall is supposed to derive any additional stability from its curvature, on the principle of the arch, as some have fancied, it must be recollected that an arch with but one abutment is a very unstable kind of structure, and such kind is simply what the curved retaining wall is. Quays and river walls may, indeed, be designed of a curved form with advantage, for such will allow of ships coming closer to the brink, than they could were the wall a straight one. And sea walls, also, are not unfrequently built of a curved section on the face, this form being under certain circumstances better adapted than a straight wall to resist the force of waves.

In situations where a retaining wall has but one purpose to fulfil—that of supporting a bank of earth—it is usual to give the base of the wall a certain amount of inclination to the horizontal, the slope being perpendicular to the batter of the face; or if the wall have a curved batter, the plane of the base will simply radiate from the centre of curvature. Such mode of construction is calculated to increase the frictional stability, for it brings the thrust of the earth from behind more nearly perpendicular to the bearing surface.

COUNTERFORTS.

Counterforts are frequently constructed at the back of retaining walls, and, although generally approved of, appear to be a somewhat doubtful mode of distributing material. Mr. Hosking, in a paper read before the Institute of Civil Engineers, deprecates their use and, with some reason, advocates the use of ribs or arches from wall to wall. These ribs seem to have been suggested by the cast-iron beams used to support the falling walls on the London and North Western Railway between Euston Station and Primrose Hill. Mr. Hosking proposes that his arches of brick should pass completely over the road, and that they should consist on plan of a pair of flat arches placed back to back. Such an arrangement would doubtless prove effective, and the expediency of adopting it would evidently be determined by the cost of the work and the value of land adjoining—a mode of construction in common use in metropolitan works, and in other situations where land is very valuable, is that shown in Fig. 4. It consists of a series

FIG. 4.



of buttresses and inverts, the convexity of which latter is opposed to the thrust of the backing. Such a distribution of material is most suitable in situations where the projection of the buttresses is not found inconvenient. In quay and river walls it would not answer of course to have any such projection, as the near approach of ships and boats is an essential consideration.

The distribution of the material in the form of counterforts is attended with a slight saving, and where buttresses would be inadmissible on account of their encroaching on the roadway, counterforts may be adopted. They have at least one use, that they oppose more friction to the earth than a plain wall, and, being easy of construction, are productive of but little additional expense. In order to ascertain what additional mean thickness a wall derives from the counterforts, it is only necessary to *multiply the length of the counterfort by its mean width, and divide the product by the distance from centre to centre* of two counterforts. The form and dimensions of counterforts vary with circumstances, the narrow and deep disposition of the material being probably the best as a general rule. The late Lieutenant Hope, of the Royal Engineers, conducted some interesting experiments on the stability of retaining walls generally, and arrived at the conclusion that a thin wall, with frequent thin counterforts, was the best arrangement of the material.

Two points of importance relative to counterforts demand particular attention—the first, that they should be built simultaneously with the wall; and the second, that the wall should be well bonded into the counterforts, otherwise they detract from the wall's strength, instead of augmenting it. It is evident that without some special system of bond, counterforts reducing the thickness of the wall, as they are generally understood to do, must prove detrimental rather than advantageous; but if plenty of hoop iron be used, which is not usually the case, counterforts may be made to contribute in a very considerable degree to the stability of the wall. In fact, quite as much as buttresses.

MODE OF BACKING AND DRAINAGE.

That accidents frequently occur from due care not being exercised in the mode of backing-up retaining walls is undoubted, and indeed to this cause alone the majority of failures is attributable; not, as is frequently supposed, to the insufficient section of the wall. The drainage of masses of earth sustained by walls, is a matter that can only be disregarded with risk of ill consequences. It is a difficult thing to prevent surface water from finding its way into earth-work, and therefore the simplest method of dealing with it will be to provide efficient means for its escape. To this end holes or *weepers* should be left in the wall at different levels, to relieve it from pressure from behind; and in order to admit the surface water to these points of escape, it will be advisable to back up the wall with dry stone, quarry shivers, or whatever else will admit the free passage of water. If a wall be backed up in this way by a rough angular material, it will be relieved of almost all pressure from the earth. Economy will, however, generally preclude such an expedient in works of considerable extent, and then it will be necessary to form the embankment with great care, adopting every precaution to prevent the tendency of the earth to slip in the direction of the wall. It will be evident from the calculation of the pressure exerted by earth, that the less the angle of repose is, the greater will be the pressure on the wall; and, as a matter of course, any means that will tend to increase the angle of repose, will relieve the wall of a certain amount of pressure. Effectual drainage will do much towards this end; but the mode of depositing the earth will also affect the angle of natural slope in a considerable degree. The same earth under different treatment will assume different slopes; if dry, it will fall when tipped—at a low angle, but if damped, and well rammed, will adapt itself to a much higher one. It has even been found that a bank when constructed in such a manner has stood for a considerable time perfectly vertical. The best mode of backing a wall up with earth will then be, to commence depositing at the foot of the wall, and to lay the earth in layers inclining *against* the wall, as shown by the dotted lines in Fig. 3, each layer being well rammed before another is commenced. This will not only consolidate the earth, and prevent any shock that might occur from sudden settlement, but will increase the angle of repose, and give the earth a tendency to slip away from the wall, rather than towards it.

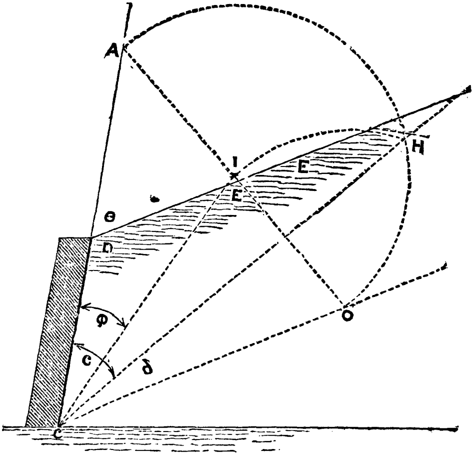
THE LAND TIE.

An expedient for securing retaining walls that is simple and not expensive, is the land tie; it consists of an iron plate, with a rod passing through its centre, the plate being placed vertically in the bank behind the wall, and the end of the rod passed through the wall and secured. The holding power will depend on the area of the plate, and the depth at which it is sunk beneath the surface. But it is evident that, in order to act most effectually, land ties should be attached to the wall at the height of the centre of pressure.

NOTE I.

The following construction, given by Mr. Neville in the "Transactions of the Institute of Civil Engineers, Ireland," vol. i., shows the method of determining the pressure exerted by a bank, whatever may be its inclination :

Fig. 5.



Let C D represent the wall; D E the face of the bank sloping at any angle; and C H the line of natural slope. Draw any line perpendicular to the line C H, cutting the line of the wall produced at A, and also a line drawn parallel to the face slope at O. On A O describe a semicircle. From

O, as a centre with the radius O H, describe an arc cutting A O in I: draw I C. The triangle C D F represents the maximum to be resisted. The angle $\delta = \theta - c$. The complement of the angle of repose $= \rho$; and the face C D $= h$

$$\tan.\,(c-\phi)=(\tan.^2\,\delta+\tan.\,c\,\tan.\,\delta)^{\frac{1}{2}}-\tan.\,\delta. \quad \text{(A)}$$

Putting R for the maximum *horizontal* resistance, and w for the weight of a cubic unit of the bank, the resistance of pressure will be

$$R=\frac{w\,h^2}{2}\,\frac{\tan.\,\theta\,\tan.^2\,(c-\phi)}{\tan.\,\theta-\tan.\,c.} \quad . \quad . \quad \text{(B)}$$

in which the value $(c-\phi)$ found above must be substituted. When C D E is a right angle we shall have

$$R=\frac{w\,h^2}{2}\,\tan.^2\,\tfrac{1}{2}\,c \quad . \quad . \quad \text{(C)}$$

the equation given in the first part of this article; and that which holds good when the slope of the bank is at right angles to the face of the wall.

The following tables calculated by Mr. J. H. E. Hart, Executive Engineer of the

Bombay Department of Public Works, are, by his kind permission, appended to this pamphlet, and will be found very convenient for the calculation of Retaining Walls.

Knowing the angle of repose of the earth to be supported, and the relative weights of the masonry of the wall and of the earth per cubic unit, a simple reference to the table will give a coefficient, which multiplied by the height will give the requisite thickness. For example, supposing a *horizontal* topped bank has to be supported by a masonry wall of 10 ft. high, and of twice the specific gravity of the earth, the angle of repose of the latter being 35 deg. Under the fraction $\frac{1}{2}$, and opposite to 35 deg., will be found in Table A the fraction .212, which multiplied by 10, the height of the wall, gives 2.12 ft., the required mean thickness.

TABLE A.

Table of coefficients of h for finding the Thickness of Standard Rectangular Walls, when the top of the bank is horizontal.

Ratios of W to W′.	$\frac{1}{1}$	$\frac{1}{1\cdot1}$	$\frac{1}{1\cdot2}$	$\frac{1}{1\cdot3}$	$\frac{1}{1\cdot4}$	$\frac{1}{1\cdot5}$	$\frac{1}{1\cdot6}$	$\frac{1}{1\cdot7}$	$\frac{1}{1\cdot8}$	$\frac{1}{1\cdot9}$	$\frac{1}{2}$	$\frac{1}{2\cdot1}$	$\frac{1}{2\cdot2}$	$\frac{1}{2\cdot3}$	$\frac{1}{2\cdot4}$	$\frac{1}{2\cdot5}$
Angle of Repose.							K	K	K							
O	.577	550	.527	.506	.488	.472	.456	443	.430	.419	.408	.399	.389	.381	.373	.365
30°	.334	.317	.304	.292	.282	.272	.263	.256	.248	.242	.235	.230	.224	.220	.215	.211
31°	.327	311	298	.286	.276	.267	.258	.251	.243	.237	.231	.225	.220	.216	.211	.207
32°	.320	305	.292	.281	.270	.261	.253	.245	.238	.232	.226	.221	.216	.211	.206	.202
33°	.314	299	.286	.275	.265	.256	.248	.240	.234	.228	222	.217	.212	.207	.203	.198
34°	307	.293	.280	.269	.260	.251	.243	.236	.229	.223	.217	.212	.207	.203	.198	.194
35°	300	.286	.274	.263	.254	.245	.237	.230	.224	.218	.212	.207	202	.198	194	.190
36°	.295	.280	.268	.258	.248	.240	.232	.225	.219	213	.208	.203	.198	.194	.190	.186
37°	.288	.274	.262	.252	.243	.235	.227	.221	.214	.209	.203	.199	.194	.190	.186	.182
38°	.282	.268	.257	.246	.238	.230	222	.216	.209	.204	.199	.194	.189	.185	.182	.178
39°	.276	.262	.251	.241	.233	.225	.217	.211	.205	199	.194	.190	.185	.180	.177	.174
40°	269	.256	.245	.235	.227	.219	.212	.206	.200	.195	190	.185	.180	.177	.173	.170
41°	263	.250	.240	.230	.222	.215	.207	.202	.196	.192	.186	.181	.177	.173	170	.166
42°	.257	.244	.234	.225	.216	.210	202	.197	.191	.186	.181	.177	.173	.169	.165	.162
43°	.251	.239	229	.220	.212	.205	.198	.192	.187	.182	.177	.173	.169	.164	.162	.158
44°	.245	.233	.223	.214	.206	200	.193	.188	.184	.177	.172	.169	.164	161	.158	.154
45°	239	.227	.218	.209	.202	.295	.188	.183	.178	.173	.168	.165	.161	.157	154	151

TABLE B.

Table of coefficients of h for finding the Thickness of Standard Rectangular Walls, when the top of the bank slopes away at the Angle of Repose.

Ratios of W to W′.	$\frac{1}{1}$	$\frac{1}{1\cdot 1}$	$\frac{1}{1\cdot 2}$	$\frac{1}{1\cdot 3}$	$\frac{1}{1\cdot 4}$	$\frac{1}{1\cdot 5}$	$\frac{1}{1\cdot 6}$	$\frac{1}{1\cdot 7}$	$\frac{1}{1\cdot 8}$	$\frac{1}{1\cdot 9}$	$\frac{1}{2}$	$\frac{1}{2\cdot 1}$	$\frac{1}{2\cdot 2}$	$\frac{1}{2\cdot 3}$	$\frac{1}{2\cdot 4}$	$\frac{1}{2\cdot 5}$
Angle of Repose.							K	K	K							
30°	.500	.476	.456	.438	.423	.409	.395	.384	.372	.363	.353	.345	.337	.330	.323	.316
31°	494	.471	.452	.434	.418	.404	.391	.380	.368	.359	.350	.342	.333	.326	.320	.313
32°	489	.466	.447	.429	.414	.400	.387	.376	.365	.355	.346	.338	330	.323	.316	.309
33°	483	.461	.442	.424	.409	.395	.382	.371	.360	.351	.342	.334	326	.319	.312	306
34°	.478	.456	.437	.419	.404	.391	.378	.367	.356	.347	.338	.331	.322	.316	.309	.303
35°	.472	.450	.431	.414	.399	.386	.373	.362	.352	.343	.334	.327	318	.312	.305	.299
36°	.467	.445	.426	.409	.395	.382	.369	.358	.348	.339	.330	.323	315	.308	.302	.295
37°	.461	.439	.421	.404	.389	.377	.364	.345	.343	.334	.326	.319	.311	.304	.298	.291
38°	.455	433	415	.399	.384	.372	.359	.349	.339	.330	.321	.314	.306	.300	.294	.288
39°	448	.427	409	393	.379	.367	.354	.344	.334	.325	.317	.310	.302	.296	.290	.284
40°	.442	.421	404	.388	.374	.361	.349	.339	329	.321	.312	.306	.298	.292	286	279
41°	.435	.415	.397	.382	.368	356	.344	.334	.324	.316	.307	.301	293	.287	.281	.275
42°	.429	.409	.391	.376	.363	.351	.339	.329	.319	.311	303	.296	289	.283	.277	.271
43°	.422	402	.385	.370	357	.345	.333	.324	.314	.306	.298	.292	.284	.278	.273	.267
44°	.415	395	379	.364	351	.339	.328	.318	.309	.301	.293	.287	.280	.274	.268	.262
45°	.408	.389	.373	.358	.345	334	.322	.313	.304	.296	.288	.282	.275	.269	.264	.258

NOTE II.

The following graphic method for determining the pressure of earth against a retaining wall, we take from "Engineering:"

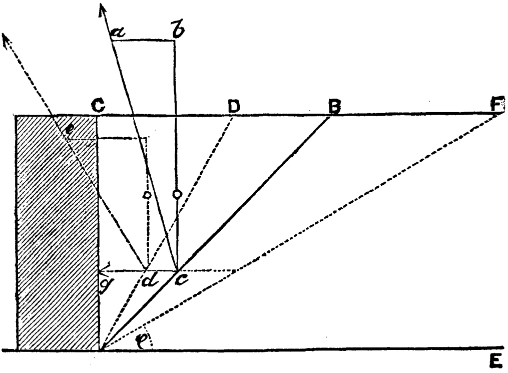
Referring to Fig. 1, let us determine, first, the pressure exerted by the wedge, A, B, C, the angle, B, A, E, being greater than Φ, the limited angle of resistance of the wedge against A, B, which is also identical with the natural slope of the earth. The friction against the back of the wall is neglected. We have now 3 forces to deal with—namely, the weight of the triangle, A, B, C, acting vertically through its centre of gravity, and therefore passing through the point c, where A, $c = \frac{1}{3}$ A, B; next, the resistance of the plane A, B, the direction of which is inclined at an angle, Φ, to the normal to A, B; and, lastly, the thrust against the back of the wall acting horizontally through c, and cutting A, C in a point, g, where A $g = \frac{1}{3}$ A, C.

Now, since the weight of the wedge, A B C, is proportional to C B, the height of the wall remaining constant, and the slope varying, if we set off *c b* = C B, and complete the triangle of forces, *a b* will represent the thrust against the back of the wall.

Let us now observe the effect produced by altering the slope A B, to A D.

Construct the triangle of forces *d e f*, as before, making *d f* = C D, to represent

FIG. 1.

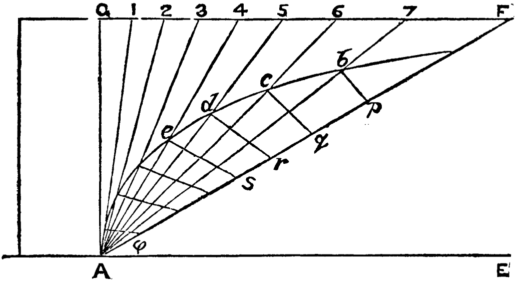


the weight of A C D. The angle, *e d f*, is now greater than the angle *a c b*, by the same amount that we have increased the slope of A B to A D; that is to say, the angle $e\,d\,f = a\,c\,b +$ angle D A B.

Also, the length of $b\,c$ has decreased to $f\,d$ in the ratio of C B to C D.

Supposing now we divide up the angle C A F into any number of equal angles by the radial lines A_1 A_2 A_3, etc. (see Fig. 2), and imagine the slope of A B to be altered

FIG. 2.



to each of these positions successively, we shall then for each alteration have a new triangle of forces; for instance, in moving from the position A_1 to A_2 the angle b A p, of the triangle of forces will increase to c A q in the same ratio that the slope of the plane varies, and the side A *b* will decrease to A *c* in the same ratio that C_1 decreases to C_2.

We are thus enabled to make a diagram illustrating the successive changes by a curve, *e d c b*, A *b*, A *c*, A *d*, etc., being respectively equal to C_1, C_2, C_3, etc. The lines *b p*, *c q*, *d r*, *e s*, and *c* drawn at right angles to A *b*, A *c*, A *d*, etc., will now represent the thrusts against the back of the wall at the different slopes, and it will be observed on examining the diagram that the position which gives the greatest magnitude to the line representing the thrust is the slope A_4, which bisects the angle C A F.

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